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CHAPTER 6
FREE-STANDING TOWER STUDIES

6.1   INTRODUCTION AND OBJECTIVES

The objective of the freestanding tower studies was to measure the response of a freestanding tower to extreme winds. The freestanding tower is particularly vulnerable to wind loads after it is completed, during the construction phase, but before it is braced by the suspension cables in the north-south direction. A fully aeroelastic, 1:250 scale model of the free-standing tower was made and tested in a 1:250 scale, modeled "coastal" atmospheric boundary layer.

6.2   MODELED WIND ENVIRONMENT

The 1:250 scale "coastal" atmospheric boundary layer used for the full-bridge model was used for the freestanding tower study. See Section 5.2.

6.3   FREE-STANDING TOWER MODEL

The free standing tower model described in Section 5 was used for the freestanding tower study. The aluminum spine stiffnesses and tower masses were adjusted to satisfy Froude number modeling. Therefore, the dynamic response was expected to be similar to the full-scale response, without adjustment. Translations of the full-scale tower were expected to be 250 times the model translations that were measured; and full-scale tower rotations were expected to equal the tower rotations that were measured.

6.4   EXPERIMENTAL PROCEDURES

Tower top displacements (NS motions of each leg and the EW motion of the pair of legs) were measured with Schaevitz 050-HR LVDT transducers. Lightweight wooden connecting rods and lightweight cores with a total mass of 2 g for each transducer were used. The tributary tower mass for each transducer was approximately 200 g. The modeled tower, and transducer connections are shown in Figure 5.4.

The model was disturbed by hand and the decaying free vibrations were recorded, in the fundamental NS rocking mode, the fundamental EW rocking mode, and the fundamental torsional mode about a vertical axis. Natural frequencies and damping ratios were obtained from the properties of a theoretical single-degree-of-freedom free vibration that fit best to one of the modal responses. Each mode was excited separately for this analysis.

Tower motions, in each of the three modes of vibration, were recorded for winds coming from all directions (in increments of 15 degrees), for full-scale equivalent wind speeds ranging from 0 to 50 m/sec (one hour averaged wind speed at a reference elevation of 50 m). Motions were

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recorded for a 30 second sample at a rate of 200 samples per second. For each coordinate direction a mean response, a maximum response, a minimum response, and a standard deviation response away from the mean response were computed. All displacements were converted to full-scale equivalent responses.

6.5   RESULTS OF THE FREE-STANDING TOWER STUDY

Positive coordinate directions are shown in Figure 6.1. Full-scale equivalent tower top displacements for winds from 0, 15, 30, 45, 60, 75, and 90 degrees away from the NS axis are shown on Figures 6.2 through 6.8.

Of particular interest is the vortex induced response, at a wind direction of 90 degrees (EW winds), at a wind speed of 12.67 m/sec, of +/- 29.0 cm in the Y-direction (see Figure 6.8). This response does not occur for winds 75 degrees off of NS, but only 90 degrees. That suggested that a single tower leg by itself was not particularly vulnerable to vortex induced excitation. On the other hand, when the two legs were aligned, an interaction between the wake shed from windward tower and the wake shed from leeward tower must have occurred that produced a significant and amplified response.

Normally, a large response for a single wind speed from a single wind direction would not be a major concern except that, during the Spring, Summer, and Fall from about 2:00 pm to 6:00 pm every day, the wind is expected to blow steadily from the West at Carquinez Straits, at a wind speed of approximately 12.67 m/sec (28 mph) at the bridge deck level. It is for others to determine whether or not a peak tower top displacement of +/- 29 cm (+/- 11.4 in) is a structural concern. The corresponding peak accelerations associated with this vibration are +/- 9.5% g, which certainly is a concern from an operational point of view (worker discomfort and safety).

Spoilers (on the windward and leeward legs, for the upper two-thirds of the towers) were added as shown on Figure 6.9. These spoilers reduced the response somewhat (to a peak response of approximately +/- 22 cm). The ensuing responses were still at an unacceptable level. Previous studies on multiple power plant stacks aligned also showed that spoilers (that were effective in reducing vortex induced motions of a single stack) were ineffective in reducing downstream stack motions. The same conclusion appears to be true for this case.

It is recommended that mechanical means be employed, during the construction phase when the towers are freestanding, to limit expected vortex induced tower motions. Possible mechanical means include guys, guys tensioned by sliding blocks of concrete on inclined ramps, tuned mass dampers (TMD's), etc.

Vortex induced motions of the towers will not occur in the final bridge configuration, after the main suspension cables have been installed.
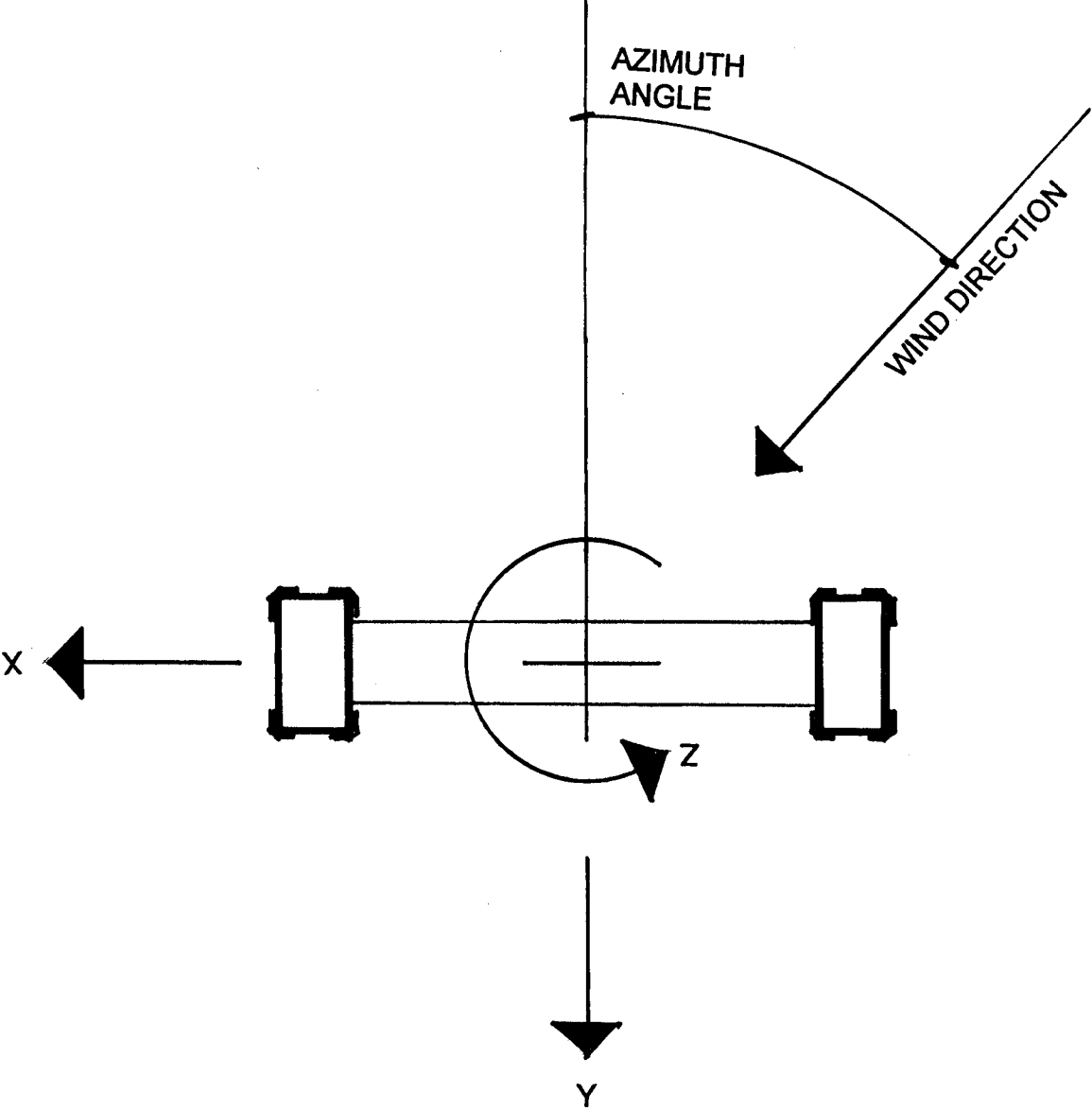
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AZIMUTH ANGLE

WIND DIRECTION

X

Z

Y

FIGURE 6.1
FREE-STANDING TOWER
POSITIVE COORDINATE DIRECTIONS

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CARQUINEZ STRAITS BRIDGE
AZIMUTH ANGLE 0
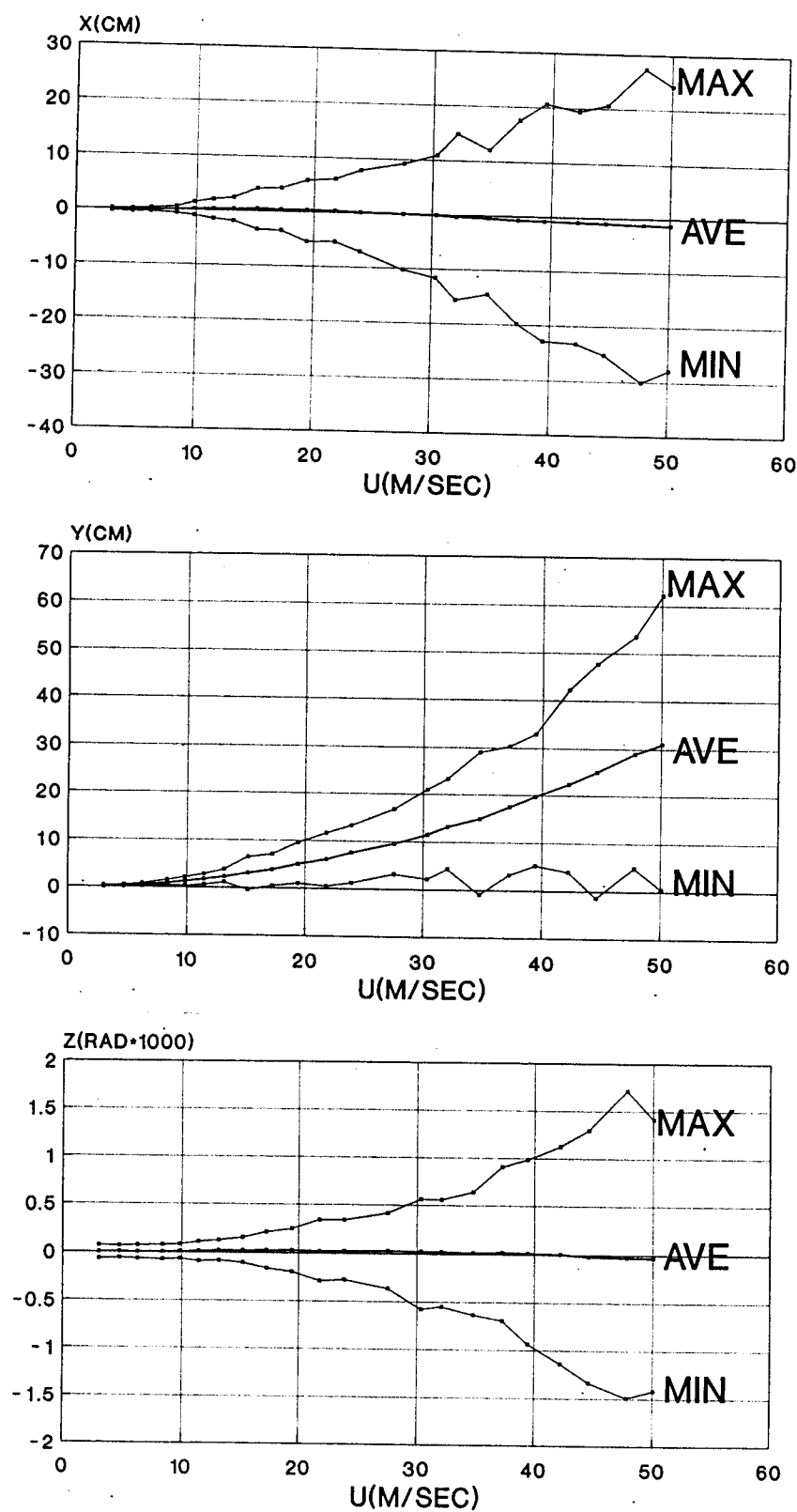
FIGURE 6.2

CARQUINEZ STRAITS BRIDGE
AZIMUTH ANGLE 15

X(CM)

U(M/SEC)

Y(CM)

U(M/SEC)

Z(RAD•1000)

U(M/SEC)

FIGURE 6.3

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CARQUINEZ STRAITS BRIDGE
AZIMUTH ANGLE 30

X(CM) vs U(M/SEC) — curves labeled MAX, AVE, MIN

Y(CM) vs U(M/SEC) — curves labeled MAX, AVE, MIN

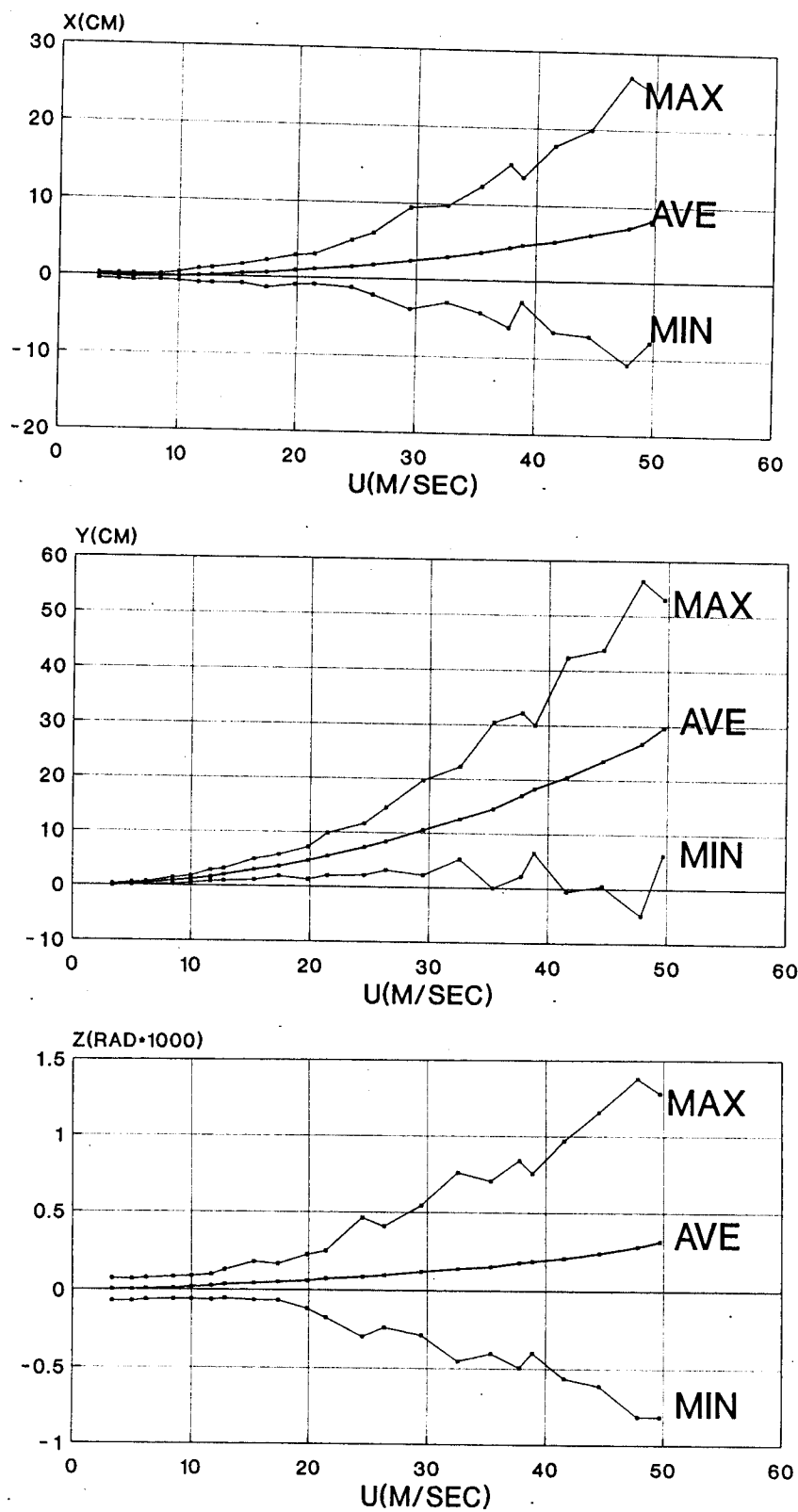Z(RAD·1000) vs U(M/SEC) — curves labeled MAX, AVE, MIN
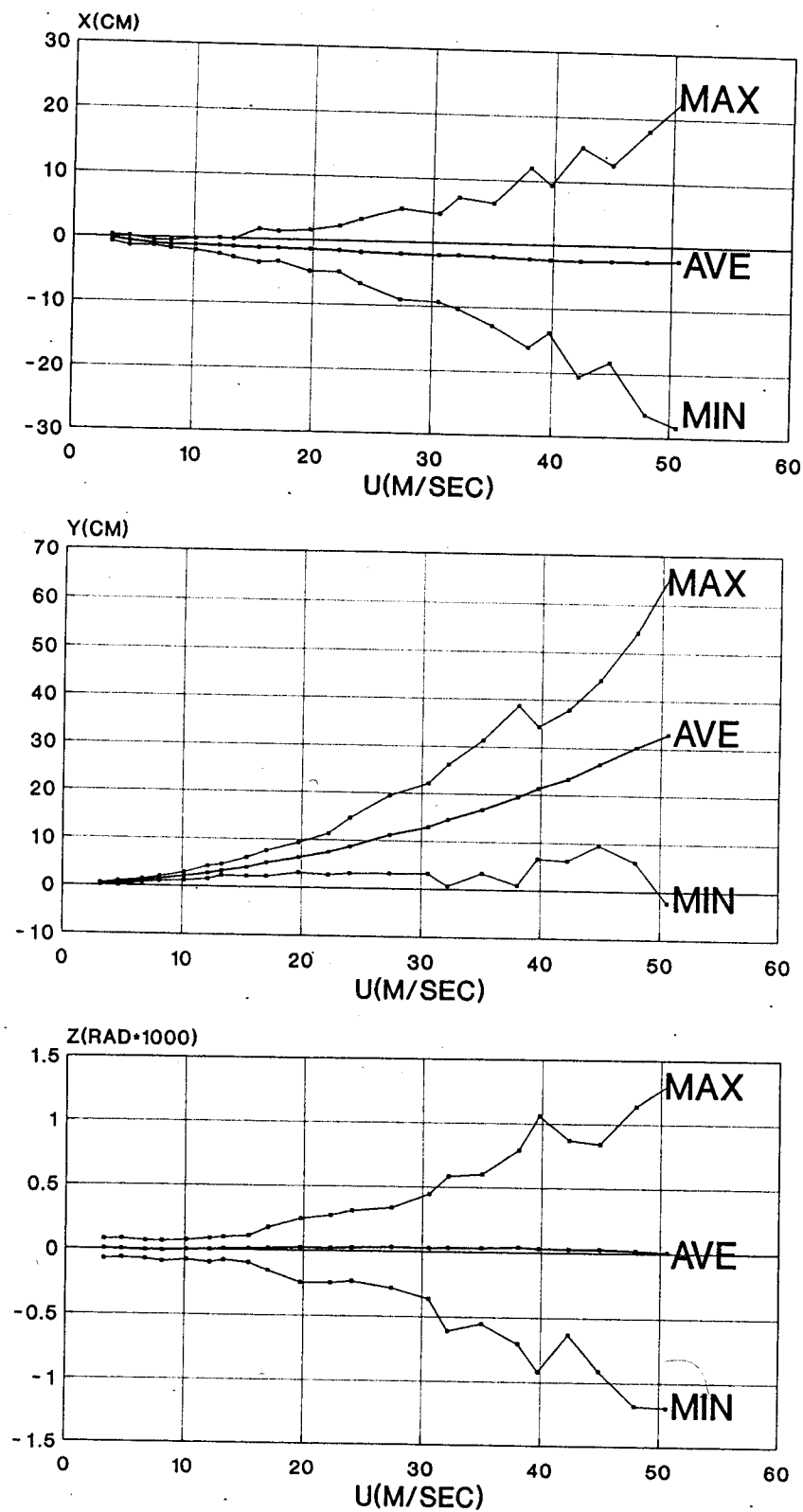
FIGURE 6.4

CARQUINEZ STRAITS BRIDGE
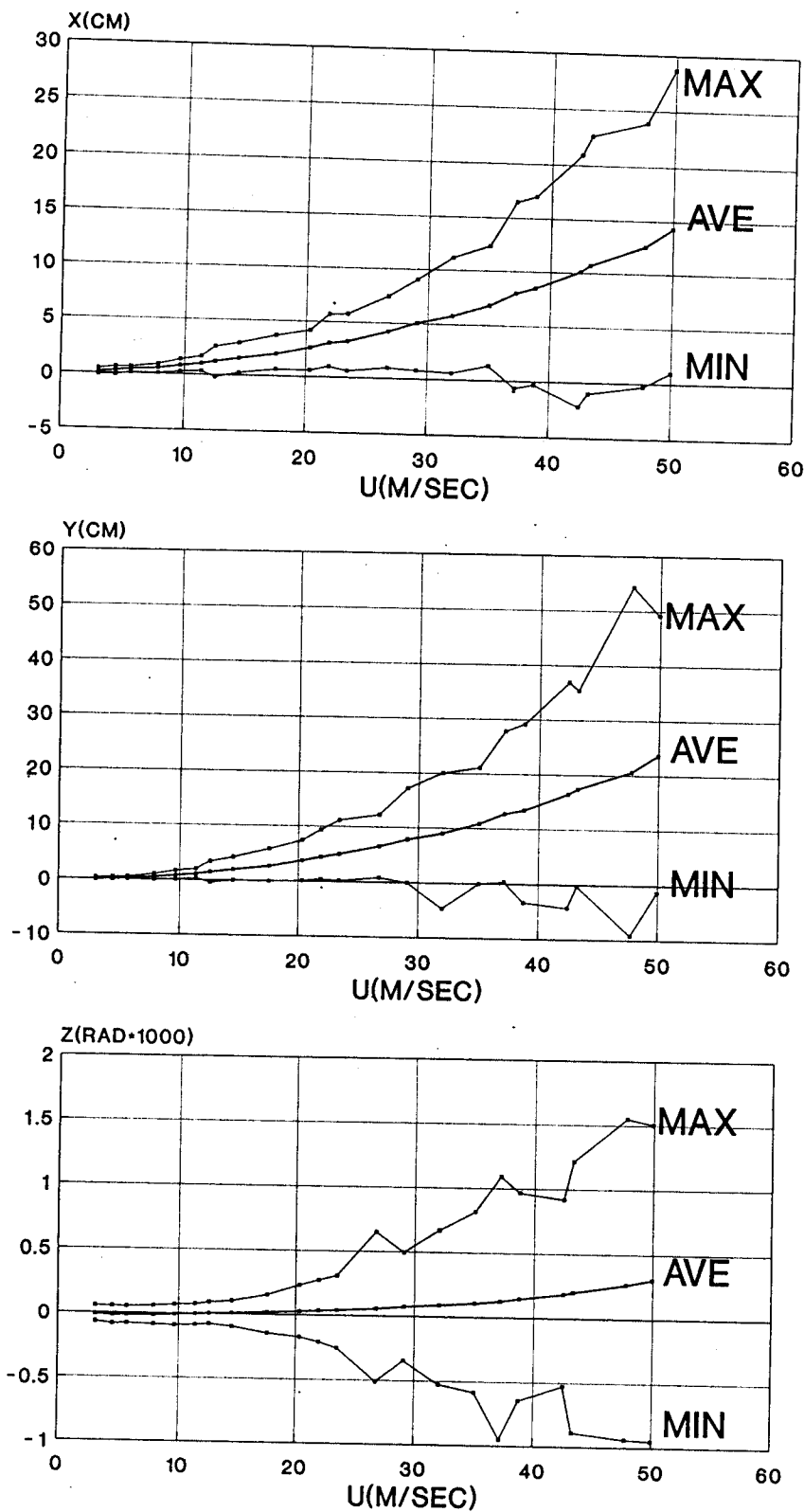### AZIMUTH ANGLE 45

FIGURE 6.5

CARQUINEZ STRAITS BRIDGE
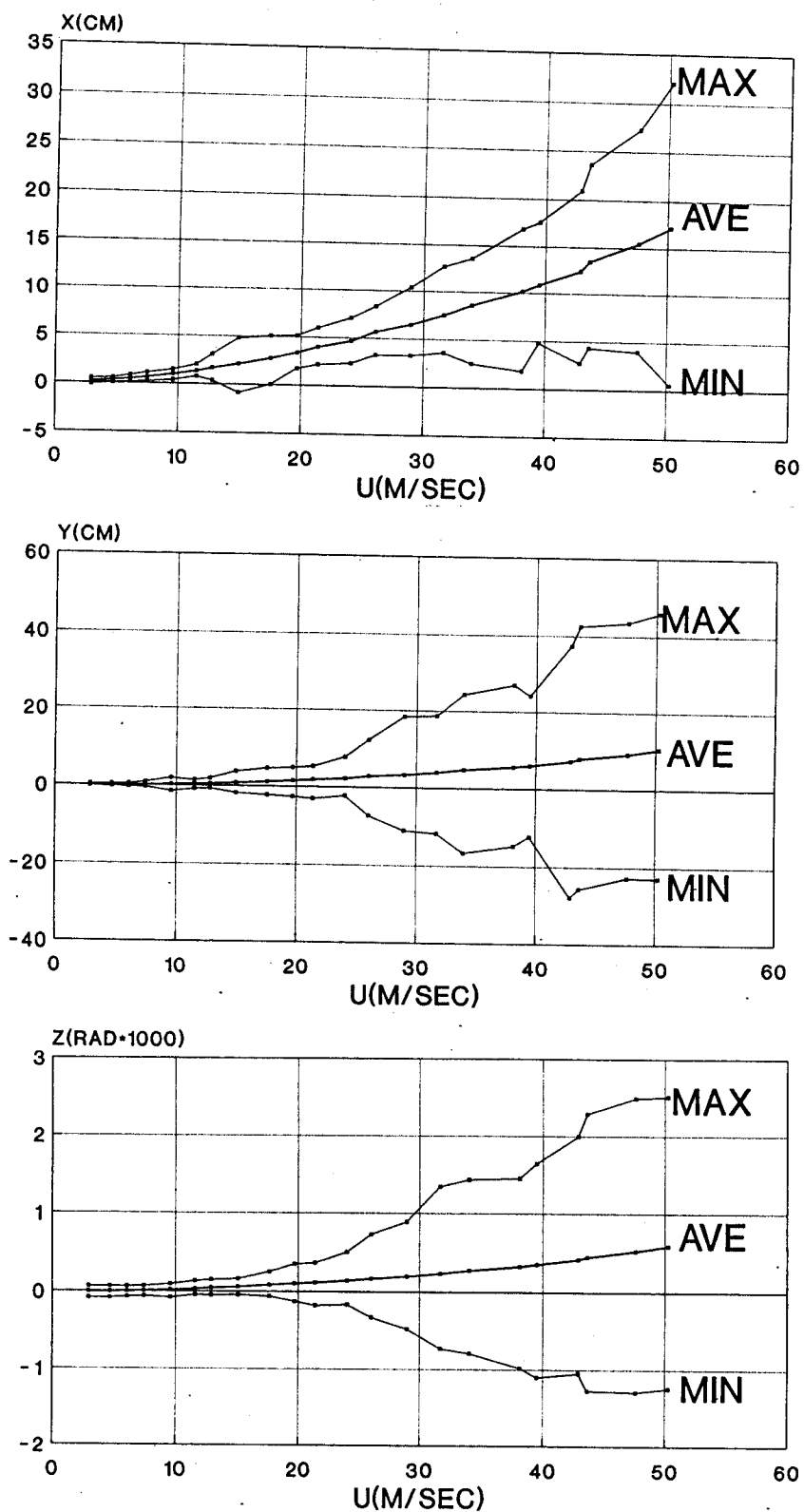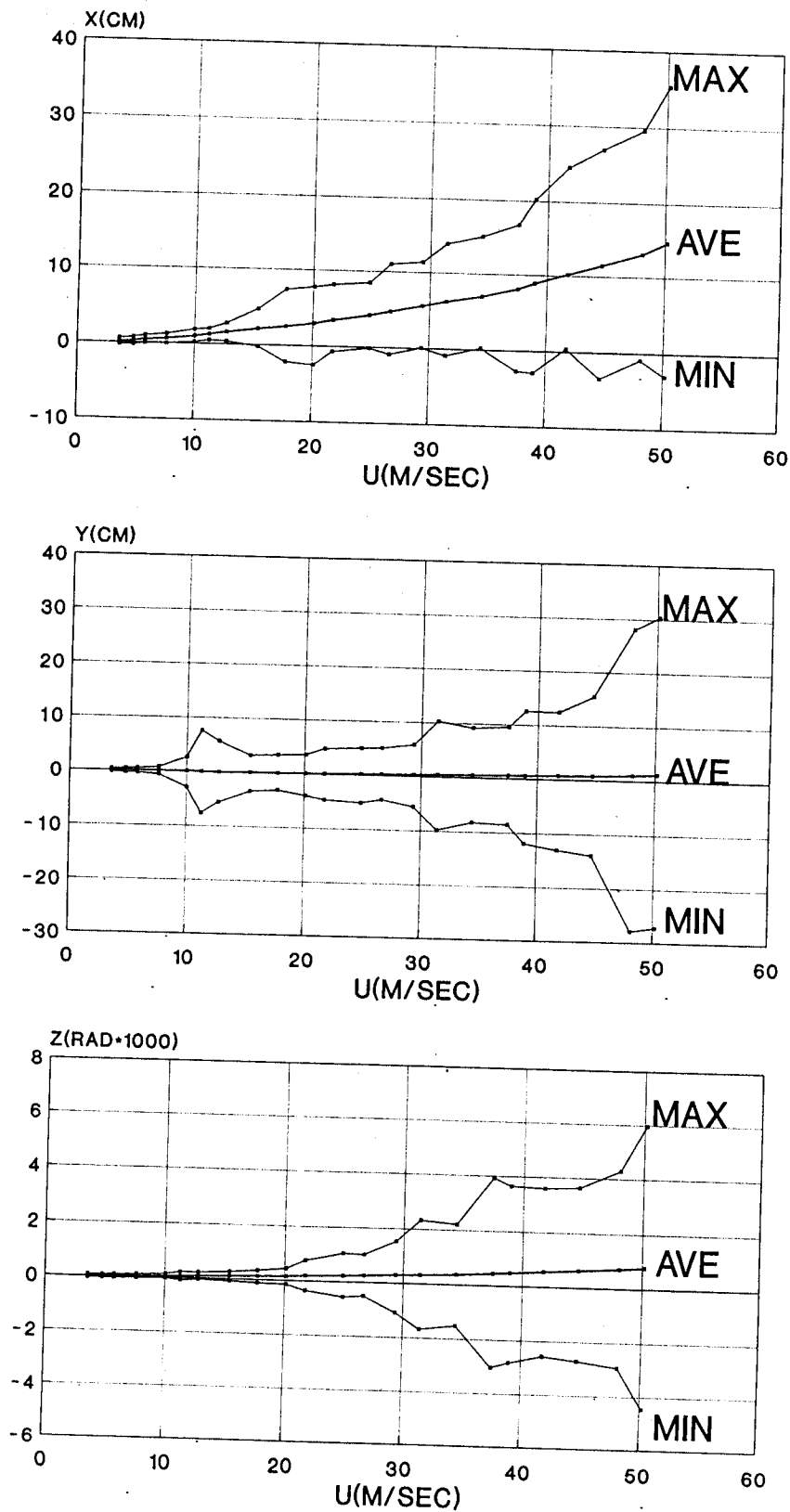## AZIMUTH ANGLE 60

FIGURE 6.6

CARQUINEZ STRAITS BRIDGE
AZIMUTH ANGLE 75

FIGURE 6.7

CARQUINEZ STRAITS BRIDGE
AZIMUTH ANGLE 90

FIGURE 6.8

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VARIES

2.44

4.88

2.44

SPOILER A

VARIES

7.32

2.44

SPOILER B

VARIES

1.22
DEEP
STRAKE

SPOILER C
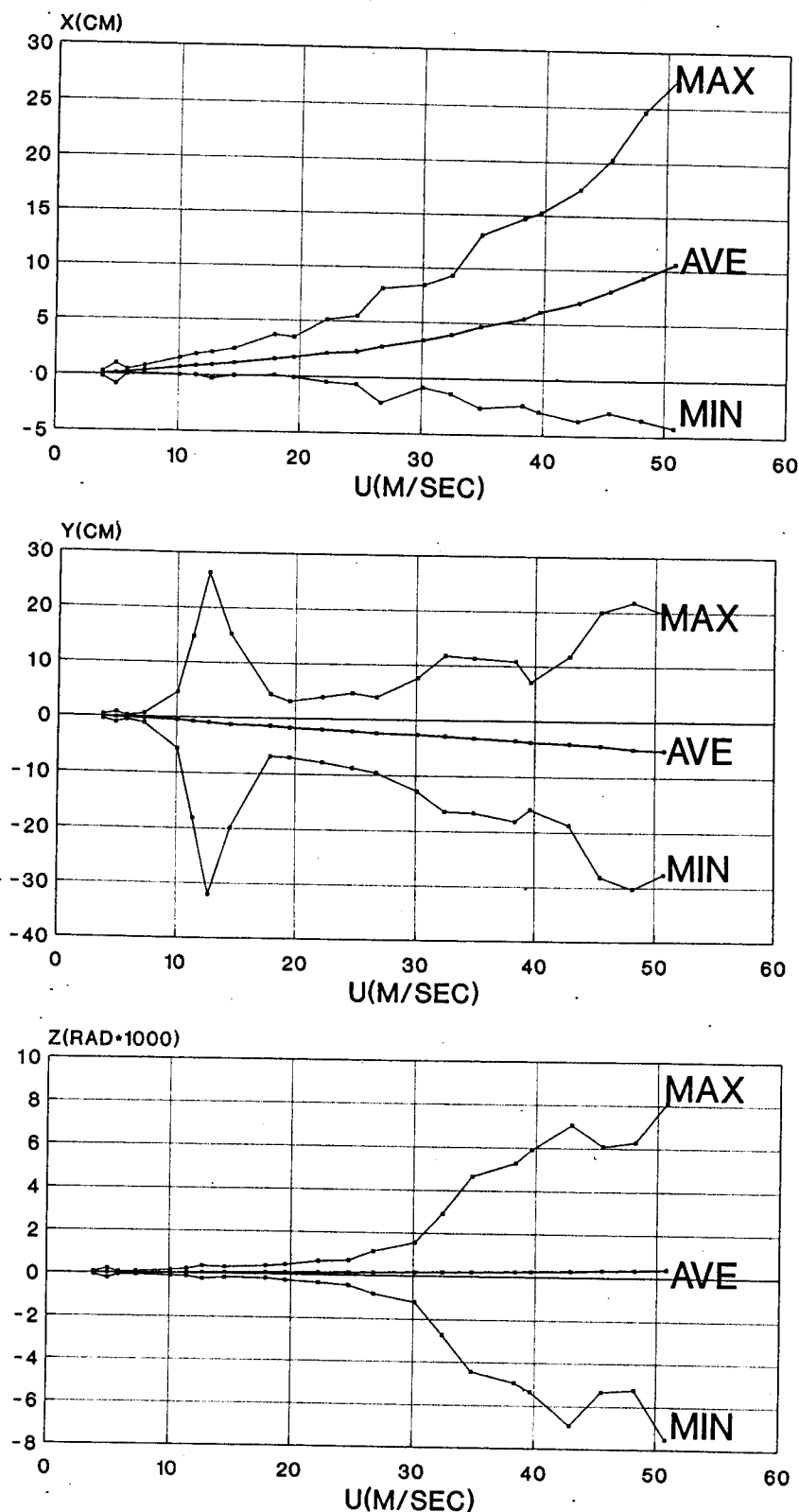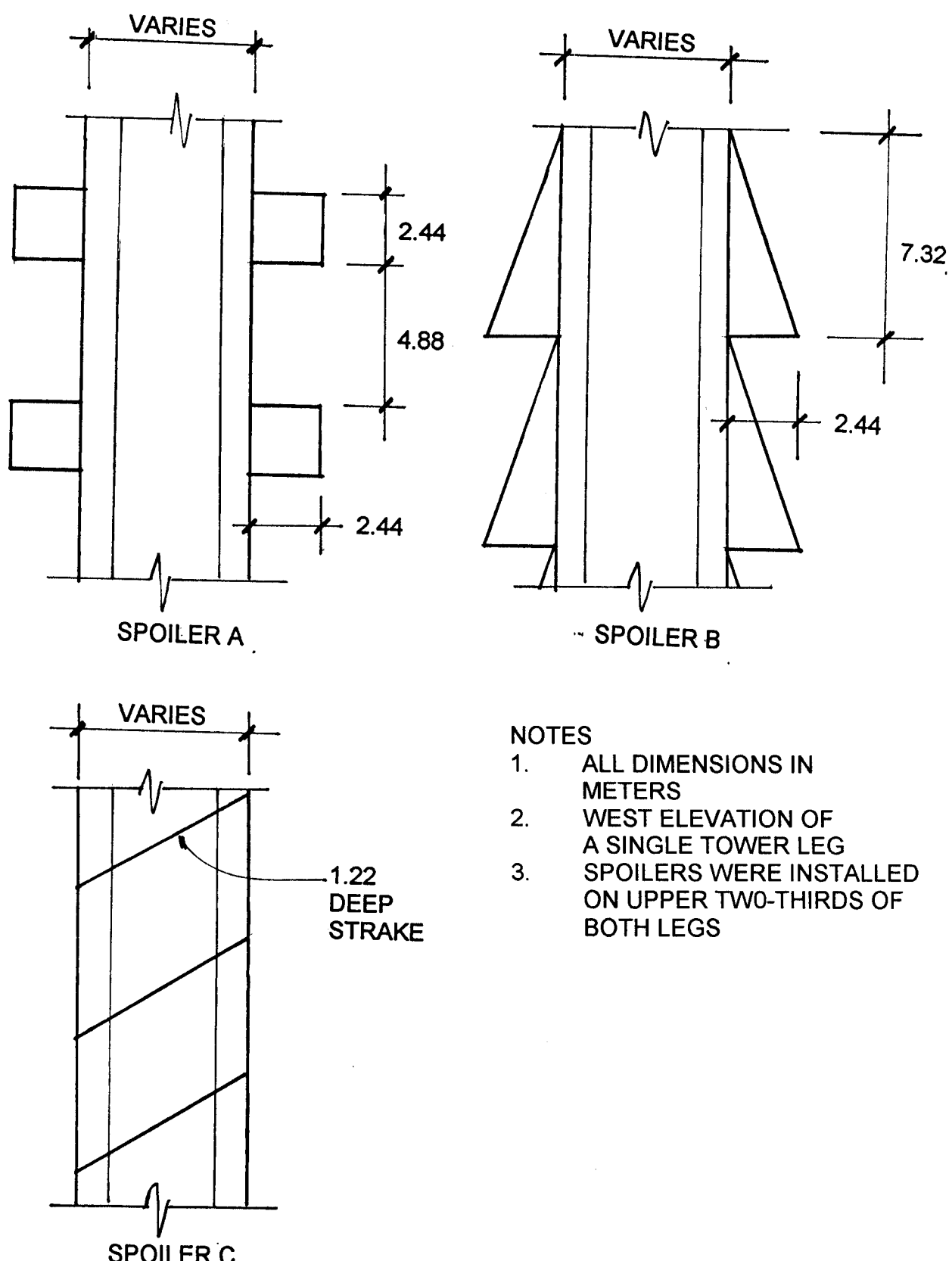
NOTES
1. ALL DIMENSIONS IN METERS
2. WEST ELEVATION OF A SINGLE TOWER LEG
3. SPOILERS WERE INSTALLED ON UPPER TWO-THIRDS OF BOTH LEGS

FIGURE 6.9
SPOILER OPTIONS

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CHAPTER 7
MISCELLANEOUS WIND RELATED STUDIES

7.1 DRAG ON TOWER LEGS

The drag coefficient on the tower legs was originally estimated (by the West wind Laboratory, Inc.) to be 1.6. A more detailed wind tunnel test was performed to measure drag coefficients on the tower legs more precisely.

A 1:50 scale model of a typical tower leg was constructed of hardwood. Wind pressures were measured in the 0.46 x 0.46 m wind tunnel (see Figure 3.2) with a Setra System, Inc. 239 Pressure Transducer. The transducer has a maximum range of +/- 34.85 N/m^2. One minute averaged pressures were measured at 30 locations around the section as shown in Figure 7.1. Pressures were reduced to a dimensionless pressure coefficient, $C_p = P / (1/2\rho U^2)$, where U is the mean wind speed at the model centerline (approximately 15 m/sec). Pressures were numerically integrated over the entire model surface to obtain a total drag force. Overall drag coefficients were obtained from

$$C_D = D / 1/2\rho U^2 B)$$

where

D   total drag force per unit length; and

B   projected width of the tower leg perpendicular to the direction of the wind.

For winds in the NS direction, the drag coefficient on a typical tower leg was found to be $C_D = 1.0$. For winds in the EW direction the drag coefficient on a typical tower leg was found to be $C_D = 1.3$.

7.2 LONGITUDINAL AERODYNAMIC DRAG ON THE DECK

Experimental wind tunnel tests to determine the longitudinal drag force on the deck were not made. Instead, a longitudinal drag force was estimated on a component by component basis.

Elements included in the drag calculation were the following: hanger supports, vertical railing elements, rail supports, and lamp posts. Also included was friction drag on all flat surfaces. A surface friction coefficient of 0.00175 was assumed based upon pipe friction coefficients at high Reynolds Numbers. A drag coefficient of 2.0 was assumed for flat plate elements, positioned perpendicular to the wind.

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Per meter of length of bridge deck, the longitudinal drag can be given by (allowing some shielding from railing vertical to railing vertical)

$$F_L = (1/2\rho U^2)(4.5152)$$

Referring both to the reference length B = 27.2 m, the C_D (LONG) = 0.0338 and C_D (TRANS) = 0.166. Longitudinal drag is approximately 20% of transverse drag.

For winds other than transverse or longitudinal, it is reasonable to assume that the drag varies smoothly from 0.166 to 0.0338. It is reasonable to assume that this smooth transition can be described by an elliptic function. Specifically

$$F(\varphi) = ((F_T Cos\varphi)^2 + (F_L Sin\varphi)^2)^{1/2}$$

where φ is the angle of the wind away from the EW axis.

7.3   GUST RESPONSE FACTOR

The dynamic response of a flexible structure can sometimes be described conveniently in terms of an equivalent set of static loads that produce a response equal to the actual peak dynamic response. For transverse sway, and overall lateral loading on the entire bridge, it was estimated preliminarily that the total lateral wind load could be given by the summation over all elements

$$LOAD = \sum (1/2\rho U^2)(C_D)(A)(GF)$$

where

C_D is the drag coefficient contributing to overall bridge lateral load for a specific element in question

A   projected area of that element

U   one hour averaged reference wind speed (47 m/sec at z = 50 m)

and

GF   gust response factor, initially assumed to be 1.9.

Full bridge model test results in overall transverse bridge sway confirmed the validity of the gust response factor of 1.9. The detailed dynamic buffeting analysis confirmed that this gust response factor was slightly conservative by approximately 10%.

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It should be cautioned that this gust response factor should be used, in association with one hour averaged wind speeds only, and it is appropriate ONLY for overall lateral load on the entire bridge structure and for lateral sway of the bridge. Other gust response factors would be appropriate for vertical loads on the deck, deck torsion, deck bending, etc., etc. Since a complete dynamic buffeting analysis for extreme winds has been performed, all of those peak stresses (peak displacements) can be obtained directly from that dynamic analysis. The dynamic analysis is the final, exact analysis. Any gust factor approach, if used, to be valid must include all of the extreme results of the dynamic analysis. Where there are discrepancies between the two approaches, results from dynamic analysis always take precedence over the results from the approximate gust factor approach.
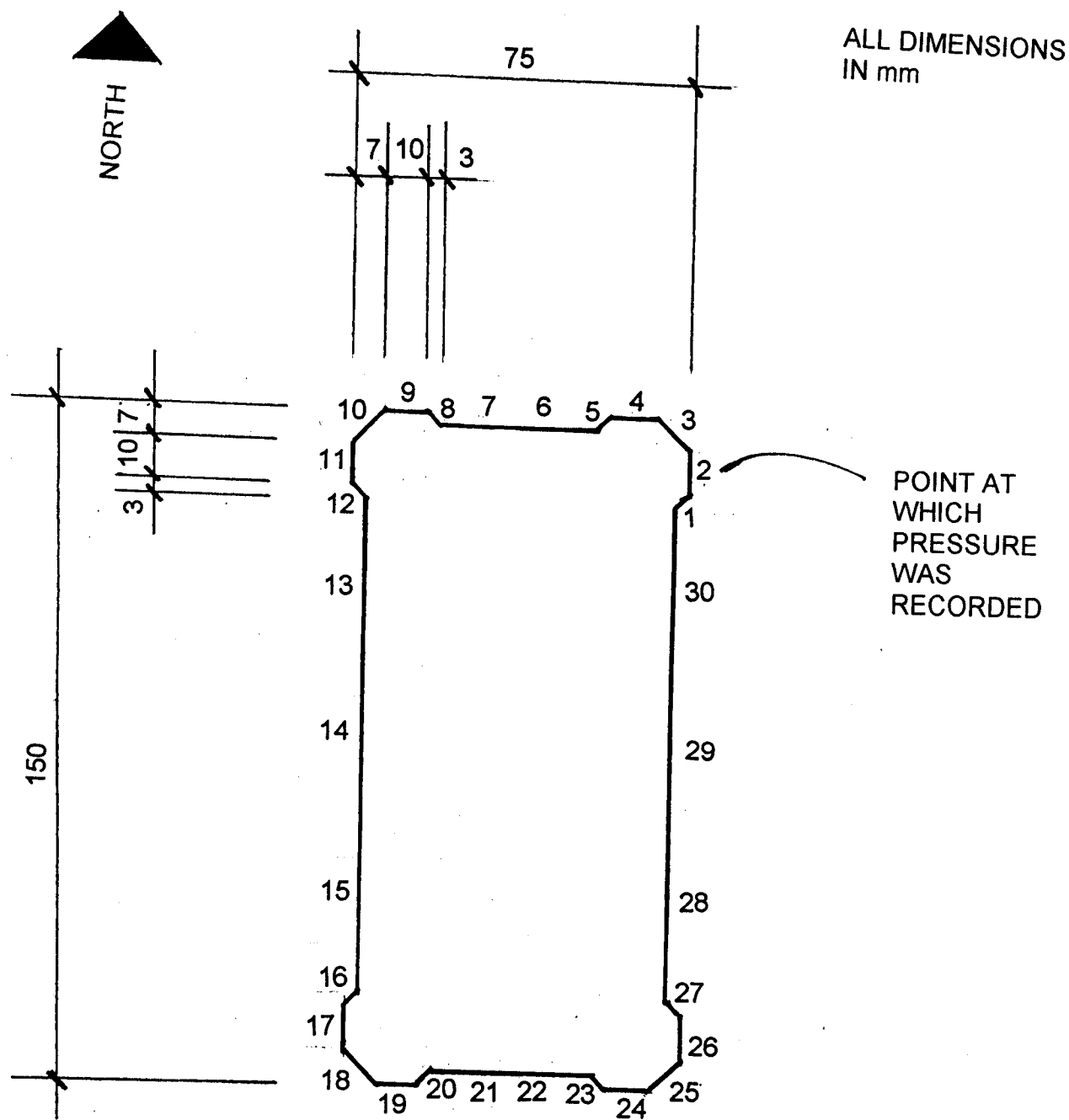
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NORTH

75

7 | 10 | 3

ALL DIMENSIONS
IN mm

7
10
3

150

10 9 8 7 6 5 4 3

11   2

12   1

13   30

14   29

15   28

16   27

17   26

18   25
   19 20 21 22 23 24

POINT AT
WHICH
PRESSURE
WAS
RECORDED

FIGURE 7.1
TOWER LEG MODEL

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REFERENCES

1. Simiu, E., and Scanlan, R. H., *Wind Effects on Structures*, Third Edition, John Wiley & Sons, New York, 1996.

2. ____, *ASCE STANDARD, American Society of Civil Engineers, Minimum Design Loads for Buildings and Other Structures, ANSI/ASCE 7-88,* American Society of Civil Engineers, New York, 1990.

3. ____, *ASCE STANDARD, American Society of Civil Engineers, Minimum Design Loads for Buildings and Other Structures, ASCE 7-95,* American Society of Civil Engineers, New York 1995.

APPENDIX A   Identification Procedure For 8 Flutter coefficients From Section Model Tests In Two-Dimensional Flow.